MARINE TECHNOLOGY AND MANAGEMENT GROUP PROJECT

Development and Verification of Computer Simulation Models for Evaluation of Siting Strategies and Evacuation Procedures for Mobile Drilling Units in Hurricanes

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CHAPTER 1

INTRODUCTION

1.1 Problem Statement

Experience with Mobile Offshore Drilling Units (MODUs) in recent hurricanes in the Gulf of Mexico (GOM) has indicated the need to reassess where and how these units are sited.

In September 1992, hurricane Andrew swept through the eastern portion of the Gulf of Mexico (GOM). Five MODUs experienced damage and inflicted significant damage on surrounding facilities. The Zane Barnes, Zapata Saratoga, and Treasure 75 all moved very significant distances during hurricane Andrew. The storm snapped seven of the semi-submersible drilling unit Saratoga's eight anchor chains and drove the unit some 100 miles to the north, where it collided with several platforms. The Zane Barnes broke loose from its eight anchors, drifted northwest some 30 miles, colliding with several platforms and many pipelines. The anchors from the Treasure 75 reportedly collided with several platforms. The LOOP (Louisiana Offshore Oil Port) 36 inch diameter pipeline narrowly missed being snagged by the dragging anchors of one of these MODUs.

This is not the first hurricane to cause such damage. The destruction of the West Delta Block 134A platform by the semi-submersible drilling unit "Blue water I" during hurricane Betsy is one of the most notable of these earlier experiences. During the approach of hurricane Betsy, the Blue water I semi-submersible drilling unit broke from its mooring and was driven by the strong winds and currents some 25 miles to the southeast until it
collided with the industry’s first platform installed in a water depth greater than 300 feet. Portions of the Blue Water drilling unit were found inside the destroyed platform.

This experience suggests that there are fundamental issues that need to be resolved regarding the policies and guidelines for manning, positioning and mooring semi-submersible drilling units in the GOM during the hurricane season. Special mooring areas and mooring systems need to be studied for MODU operation during hurricane season.

Also, as indicated by recent experience in Hurricane Andrew and Hurricane Juan, weather conditions can deteriorate rapidly. Timely decisions are critical to allow proper security and evacuation of MODUs. This is particularly crucial in areas with high storm intensity and low weather predictability and at a great distance from shore.

To limit the exposure of personnel and property, security and evacuation operations must be conducted within a reasonable time before storm conditions intensify to the point that evacuation would be hazardous. Developing frameworks for modeling these operation and evacuation systems for various weather conditions can assist in creating decision criteria for evacuations. Engineering analyses need to be performed to evaluate MODU security and evacuating alternatives, to develop general criteria and guidelines for evacuation planning. This work addresses the development of such a framework and evaluates various MODU securing alternatives using probabilistic risk analysis (PRA).
1.2 Research Objective and Scope

The fundamental issue addressed by this research is development of a rational and simplified method to evaluate the siting strategies and evacuation procedures for mobile offshore drilling units. The objective is to address this issue by developing and verifying such methods. There are two ways to approach such problems: 1) by analysis, and 2) by simulation. Simulation is normally employed when the problem is complex and there are few fundamental laws that can be used to predict the response of the system, and when the system components themselves are difficult to describe and represent.

An analytical method was developed by TRIDENT CONSULTANTS LTD (Trident Consultants Ltd., 1992) to solve a similar problem: estimate the probability of collision between a Floating Storage Unit (FSU) and surrounding platforms in the Gulf of Thailand (GOT). In that study, the probability that the FSU would collide with a platform was treated as the product of three other probabilities: the probability that the FSU would break its mooring, the probability that it would move in a given direction, and the probability that it would encounter a platform in its direction of travel. The detailed background of this analysis will be discussed in Chapter 4. This analysis is ideal for the problem in GOT, as the groups of platforms were all located within a 30 by 70 Nautical mile rectangle. Thus, it was relatively easy to calculate the three probabilities.

In the Gulf of Mexico there are much larger areas occupied by platforms. A MODU generally can have a long trip before it collides with platforms. Because hurricane centers
move with time, the associated wind and wave fields also change with time, and so the MODU's route will be variable. Also, there are different mooring system failure modes (e.g., mooring lines broken or anchors dragging), and different MODU moving modes (free floating or skipping). All these factors will have a great influence on the MODU's route and thus on the resulting collision probability. In this case, it is very difficult to calculate the three probabilities.

In this research, the problem is approached by direct computer simulations using probability models for the hurricane parameters and the MODU's approach to the site.

The first part of the report summarizes the development of an analytical model to evaluate MODU movements in response to the combined load effects of hurricane winds, waves and currents. In addition, a Monte-Carlo simulation process to evaluate the probability of collision between the MODU and surrounding large facilities is developed.

To evaluate MODU evacuation procedures in hurricanes, a computer simulation is clearly the best approach. McCarron (1971), Burke (1977), Hoffman (1978), Chen (1983), and Praught (1982) are examples that show industry's interests in decision-making using sophisticated computer simulation programs. The objective of the second part of the report is to summarize the models and procedures used to develop a computer simulation program, EVACSIM, to calculate the probability of safe evacuation, including the effects
of hurricane forecasting, evacuation start-times, available resources, and alternative evacuation procedures.

This report documents development and verification of computer models that can be used to simulate the MODU moving and colliding probabilities, and evacuation procedures during hurricanes.

1.3 Research Methodology

Figure 1.1 summarizes the approach used to develop, verify and implement the simulation models. Based on fundamentals of statistics, hurricane forecasting and modeling, fluid dynamics, mooring strength analysis, and Monte Carlo techniques, the first step of the research was to develop a basic simulation model MODUSIM, to evaluate the movement of MODUs in hurricanes. MODUSIM was then used to develop siting strategies for Mobile Offshore Drilling Units.

The simulation model incorporates models of hurricane winds, waves, currents, and tracks; storm wind, wave and current forces; mooring capacity characteristics, and finally, a model of movement characteristics that takes account free floating, intermittent grounding ('skipping'), collision 'holding', and anchor dragging characteristics. MODUSIM allows the user to specify hurricane characteristics in a probabilistic or deterministic manner. A Monte-Carlo simulation model is utilized to perform probabilistic calculations. The model includes a Markov model to describe the probabilities associated with changes
in the tracks of the hurricanes. The model incorporates variable hurricane parameters and their correlation, the storm spatial geometry, and shallow water shoaling effects. MODUSIM allows one to define the locations and sizes of 'critical facilities' near the MODU location, and then evaluate the probabilities of collisions between the MODU and the critical facilities. The organization and theoretical basis for MODUSIM will be detailed.

![Diagram of MODUSIM and EVACSIM processes](image)

**Figure 1.1 Interactive Development, Verification, and Calibration of MODUSIM & EVACSIM**

Then, based on the project management and network simulation techniques, a computer simulation model EVACSIM was developed to help evaluate operational and evacuation systems for MODUs in hurricanes. Probabilistic risk analysis and Monte Carlo techniques
are used in the model to include the large uncertainties in hurricane forecasts and in the evacuation process.

To begin the evacuation simulation process, first, statistical analysis of the hurricane forecast was performed to calculate the forecast percent error based on the differences between forecasts and history. Then, the critical environmental shut-down conditions, for example, wave heights no higher than 40 ft, were used to determine the workable time period for different operations and resources based on the hurricane forecast. The entire evacuation sequence, including the time needed for operations and the relationship between operations, are modeled using network simulation techniques. The duration for safe completion of the evacuation was determined. Histograms of evacuation risks for different evacuation start times were obtained.

The platform security and evacuation operations are modeled in Microsoft Project 4.0. Microsoft Visual Basic is used to generate the input file of MS Project to determine the resources workable time. Because of the different resource workable times associated to different weather conditions, the evacuation duration and cost depend on the starting time. Results using different evacuation start times are compared. Probabilities are determined for evacuation and securing risks dependent upon the storm severity and the type of operational accident. Sensitivity of the overall failure probabilities to the weather conditions is examined.
1.4 Organization of the Report

Chapter 2 contains background about hurricane wind and wave field modeling and the development of a simplified storm loading calculation procedure. The modeling of MODU capacity is also discussed in this Chapter. Chapter 3 includes the development of two new hurricane track forecasting simulation models and a hurricane strength prediction simulation model, Track Forecast Error Statistical Model (TFESM), Markov-Chain Simulation Model (MCSM) and Strength Forecast Error Simulation Model (SFESM). Detailed Monte Carlo simulation process for MODU collision probability is discussed in Chapter 4. It includes background on Monte Carlo simulation techniques, hurricane modeling, mooring failure mode modeling, MODU moving mode modeling, parametric and verification studies. Lastly, the extension of MODUSIM to simulate movements of bottom founded platforms, ex., jack-up, is presented in this Chapter. Based on project network simulation techniques, a computer simulation model of MODU evacuation procedures (EVACSIM) is reported in Chapter 5. Risk analysis of evacuation procedure has been done based on simulation results. Chapter 6 contains a summary of the developments and findings of this research. Potential future research topics are also identified and discussed in this chapter.

Appendix A and B document the computer program MODUSIM and EVACSIM. Detailed program descriptions and user manuals are contained in these two appendixes. Appendix C contains the statistic background for distribution fitting, goodness-of-fit test performed in the research.
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CHAPTER 2

HURRICANE LOADING AND MODU CAPACITY MODELING

2.1 Introduction

One important step in the simulation process is to determine the environmental loads acting on MODUs. In general, these loads are due to wind, waves, currents which are generated by hurricanes.

Due to complexity and random nature of these loads, it is difficult, if not impossible, to develop theoretical models that accurately predict these loads and their effects on offshore structures. This is why offshore engineering research has traditionally used field measurements and laboratory experiments to calibrate existing loading models. The Conoco test Structure (Bea et al. 1986) and the Ocean test Structure (Haring et al. 1979) are two platforms highly instrumented for the purpose of measuring wind and wave forces on offshore structures.

Traditionally two different approaches have been utilized to predict the hydrodynamic loads on offshore installations: deterministic and stochastic (Bea et al., 1978). The deterministic approach itself can be pseudo-static or time-dependent. The pseudo-static deterministic approach uses the maximum wave kinematics which result in maximum loads. These loads are then used to perform static structural analyses. In the time dependent deterministic approach, loads are calculated as a function of time and used to
perform dynamic structural analysis. The stochastic approach treats the loading as a random process where the loading condition is described by spectral densities. Figure 2.1 summarizes these approaches and the major steps involved in each approach. The different methods are used to perform different types of analyses from static pushover for extreme conditions to fatigue analysis for nominal conditions.

To formulate current and wave forces on offshore platforms, two areas of fundamental research need to be addressed: a) fluid mechanics of steady and unsteady flows passing a body and b) fluid motion in a wave described by wave theories (Sarpkaya et al., 1981). For an overview of historical developments in the subject of hydrodynamic loads on offshore structures and a more detailed treatment of the subject, refer to Sarpkaya and Isaacson (1981).

The purpose of this research is to develop a simple procedure to determine the aerodynamic and hydrodynamic loads acting on a MODU. In the following sections, hurricane models are discussed first, then wind loads are formulated. The fluid mechanics background that is necessary to develop a simplified load calculation approach is also discussed. Finally, a simplified load model is introduced that uses an idealized structure and Stokes fifth order wave theory to predict the wave loads acting on MODUs. This load model is verified with results from more sophisticated current and wave load generating programs commonly used in industry.
Figure 2.1: Alternative Approaches to Wave Loading Analysis (Bea and Lai, 1978)
2.2. Hurricane Models

The hurricane is assumed to travel along a straight line with a given translation speed and direction during each simulation time interval, e.g., 2 hours. The wind field is governed primarily by the three parameters: Pressure difference, radius of maximum wind speed and hurricane transition velocity, which are $\Delta P$, $R$ and $V_t$. Changes in the storm parameters after shelf edge crossing are not considered, i.e., the intensity of the hurricane is assumed to be stationary during passage over the continental shelf. The wind and wave field are based on parametric expressions derived from more sophisticated numerical models. These parametric models which were developed by Cooper (1988) give the wind velocity, significant wave height, and wave direction as functions of the hurricane parameters and the site position relative to the storm center. The current field is based on a one-dimensional numerical model which is a simplified version of the three-dimensional numerical model by Cooper. The models are briefly described in the following sections. Details can be found in Cooper (1988).

2.2.1 Wind Field

The wind speed (in m/s) and direction $\beta$ (polar angle in degree) as functions of the position relative to the storm center in polar coordinates $r$ and $\theta$ are given by:

$$W = W_w(r/R)$$ 

for $r/R > 1$ (2.1)

$$W = 1.047W_w[1 - \exp(-3.1r/R)]$$ 

for $r/R < 1$ (2.2)

in which,

$$W_w = 0.885(5.6\sqrt{\Delta P} - 0.5Rf) + V_t \cos \theta$$ (2.3)
\[ a = -0.38 + 0.08 \cos \theta \quad (2.4) \]

where, \( f \) = Coriolis parameter in rad/s, \( \Delta P \) in mb, \( V \) in m/s, and \( R \) in m

\[ \beta_{\text{A}} = \theta + \alpha + 90^\circ \quad (2.5) \]

in which \( \alpha \) is the deflection angle given by:

\[ \alpha = 22 + 10 \cos \theta \quad (2.6) \]

2.2.2. Wave Field

The parametric model for significant wave height \( H_{\text{m}} \), in meter, at a given location can be expressed as a "25 percentile rule" (Bea, 1990) or:

\[ H_{\text{m}} = 0.25V \quad (2.7) \]

In which \( V \) is the local wind speed in m/s. The equation for the average wave direction \( \phi \) (polar angle in degree) is:

\[ \phi = \alpha + a(r / R)^* + \theta - 90^\circ \quad (2.8) \]

in which

\[ a = 144 + 39 \cos \theta - 25 \sin \theta - 15 \cos 2\theta \quad (2.9) \]

\[ b = -0.08 \quad (2.10) \]

The r.m.s. errors are of the order of 10 to 20 degrees.

The equation for peak period, \( T_r(s) \), is (Noble Denton, 1991):

\[ T_r = aW^* \quad (2.11) \]

where,

\[ a = 8.0 - 3.5 \cos \theta + 2.7 \sin \theta \quad (2.12) \]
\[ b = 0.143 + 0.138 \cos \theta - 0.074 \sin \theta \] (2.13)

### 2.2.3. Current Field

The following parametric model has been developed for the expected maximum storm current velocity \( (U_\infty, \text{m/s, average velocity in upper 30-meter thick mixed layer}) \), concurrent in time and direction with the occurrence of the expected maximum wave (Bea, 1990):

\[ U_\infty = \bar{v} V_\infty \] (2.14)

Where \( \bar{v} = 0.02 - 0.03 \), \( V_\infty \) is the 10m elevation, 10 minute average wind speed at the time that the cyclone crosses the site. The current direction is assumed the same as the wind and wave direction.

### 2.2.4. Surge of Sea Surface

The surge of the sea surface due to hurricane in deep water is determined as (Bea 1988):

\[ \Delta h = 0.03H_\infty \] (2.15)

where, \( H_\infty = f(\Delta p) \), is the maximum wave height due to the hurricane in deep water.

### 2.2.5. Expected Maximum Wave Heights, Given Significant Wave Heights

The expected maximum wave height, \( H_\infty \), could be estimated from the short term wave height distribution based on 1000 waves (expressing a 3-hour duration of the maximum sea state intensity at the location) (Bea, 1990):
\[ H_* = \zeta H_0 \sqrt{\frac{\ln N}{2}} \]  

(2.16)

where \( \zeta = 0.93 \), \( V_* = 8\% \).

Thus, the expected maximum wave height could be estimated as:

\[ H_* = 1.73H_0 \]  

(2.17)

2.2.6 Shoaling Effect

Storm waves tend to be attenuated by a variety of processes as they propagate across the relatively shallow depths of the Texas and Louisiana Continental Shelves. As a wave propagates from deep to shallow water, its height and length change. The transformed wave height, \( H \), at shallow water depth relative to the original deep water wave height, \( H_0 \), can be computed from (Shore Protection Manual):

\[ \frac{H}{H_0} = \left( \frac{V_*}{V} \right)^\frac{1}{3} \left( \frac{b}{b_0} \right)^\frac{1}{3} \]  

(2.18)

Where \( V \) is the group velocity of the waves, \( b \) is the distance between pairs of adjacent wave rays, and the subscript 0 refers to deep water condition.

The term \( \left( \frac{V_*}{V} \right)^\frac{1}{3} \) is also known as the shoaling coefficient, \( k \). The shoaling coefficient is given according to linear wave theory by:

\[ K' = \left( \frac{1}{2kh} \right)^\frac{1}{3} \left( \frac{1}{\sinh 2kh} \right) \tan \h \]  

(2.19)
Where $h$ is the water depth and $k$ is the wave number. Where, $L = \frac{gT^2}{2\pi} \tanh\left(\frac{2\pi d}{L}\right)$.

Eckart (1952) gives an approximate expression for Equation (2.19), which is correct to within about 5 percent. This expression is given by:

$$L = \frac{gT^2}{2\pi} \sqrt{\tanh\left(\frac{4\pi^2 d}{T^2 g}\right)}$$  \hspace{1cm} (2.20)

$$T = 2.7\sqrt{\frac{H}{H_{\infty}}}$$  \hspace{1cm} (2.21)

K's is then given explicitly as a function of wave length and water depth.

The term $\left(\frac{b-x}{b}\right)^2$ in the shoaling equation represents the relative spacing of adjacent wave rays and is also defined as the refraction coefficient, $K_x$. Physically, the relative spacing between wave rays represents the local wave energy density. It is generally assumed that the wave energy contained between wave orthogonal is conserved as the wave front progresses. Various graphical and numerical methods are available to compute wave refraction. In this study, the graphical procedure was adopted. However, most of the wave paths were near normal to the smoothed depth contours; thus, the wave refraction effects proved to be insignificant.

Based on data from Woodward-Clyde Consultants (1982), the shoaling effect of the current is modified as:

$$U = (2.5 - \frac{1.5}{250} d)U_o$$  \hspace{1cm} (2.22)
where \( U_0 \) is the current velocity at 250 ft depth of water (Figure 2.2).

![Graph showing shoaling effect](image)

**Figure 2.2 Shoaling Effect**

The shoaling effect of the surge of the sea surface is defined as:

\[
\Delta h' = \Delta h \cdot k
\]  

(2.23)

where \( k \) is the shoaling effect parameter:

\[
K = (1 + \frac{3(300 - \Delta h)}{290})
\]  

(2.24)

and, \( \Delta h \) is the surge in deep water (\( \geq 300 \)ft).
The wind velocity in shallow water is assumed the same as in deep water. (See Figure 2.2)

2.2.7. Expected Maximum Wave Heights and Return Period

The expected maximum deep water (300 ft) wave heights in Gulf of Mexico can be determined as (Bea, 1990):

\[
H_\infty = C(\Delta P)^{\frac{1}{3}} \psi \zeta H_{\infty} \sqrt{\frac{\ln N}{2}}
\]  

(2.25)

where \[C = 4.4\] \[V_c = 6\%\]  
[\[\psi = 0.25\]  \[V_v = 10\%\] \[\zeta = 0.93\] \[V_s = 8\%\]  
[\[\Delta P = 46.38\] \[V_{\infty} = 68\%\]]

Assuming \(H_\infty\) can be characterized with a Lognormal distribution, we have,

\[
\bar{H}_\infty = \bar{C}(\bar{\Delta P})^{\frac{1}{3}} \bar{\psi} \bar{\zeta} H_{\infty} \sqrt{\frac{\ln \bar{N}}{2}}
\]  

(2.26)

\[
V_{H_\infty}^1 = V_c^1 + \left(\frac{1}{2} V_{\infty}^1\right)^1 + V_v^1 + V_s^1 + \left(\frac{1}{2} V_{\infty}^1\right)^1
\]  

(2.27)

The average return period (ARP) was computed using Equation [2.28]:

\[
ARP = \frac{1}{\lambda \{1 - F(H_{\infty})\}}
\]  

(2.28)
where $F(H_{\text{max}})$ is the cumulative percentage of $H_{\text{max}}$ values equal to or less than a given value, and $\lambda$ is the average number of important wave-generating hurricanes affecting this area each year.

With the same procedure, we can get the ARP of Maximum wind velocity and current velocity (Figure 2.3).

![AVERAGE RETURN PERIOD YEARS](image)

Figure 2.3 Environmental Loading Return Period
2.3 Hurricane Loads

There are three major hurricane loads on a MODU: wind load, hydrodynamic wave and current load (Figure 2.4).

![Figure 2.4 Loading on MODU](image)

2.3.1 Aerodynamic Loads

Wind forces acting on the exposed portions of offshore platforms are in general not as significant as wave forces acting on these structures. However, their effect has to be included in the global structural movement analyses. Wind forces are generally composed of two components: a sustained (or steady) component averaged over a longer period of time (usually over one minute) and a gust (or fluctuating) component averaged over a shorter period of time (usually less than one minute). Sustained wind velocities are used to analyze the global platform behavior and gust velocities are used to analyze the local member behavior. In this research, the dynamic aspects of wind loading are neglected. Only sustained wind velocities are used to calculate the first order of the global platform movement.
Due to surface friction, the geostrophic wind velocity is reduced in the vicinity of ocean surface. API RP 2A (API, 1993) gives the following approximation to the wind profile,

\[ u(1hr, z) = u(1hr, z_r)(z / z_r)^{m} \]  \hspace{1cm} (2.29)

where \( z_r \) denotes a reference height usually taken as 10 meters.

2.3.2 Hydrodynamic Loads

To establish the hydrodynamic loads acting on an offshore platform, the following steps need to be taken:

a) establish wave, current, and storm surge information based on site specific studies including recorded or hindcasted data,

b) use an appropriate wave theory to describe the fluid motion and water particle kinematics,

c) use a force transfer function to determine the loads acting on platform members. In the following sections, the last two steps, b and c, are described and discussed in detail.

2.3.2.1 Wave Theories

The problem of describing the wave motion has been dealt with for more than a century. Numerous text books have been devoted to development of various wave theories and describing their results (refer to Sarpkaya and Isaacson, 1981, for a comprehensive list of references). All of these wave theories are based on the following common assumptions: the waves are two-dimensional and propagate in horizontal direction in waters with
constant depth and a smooth bed. It is further assumed that the wave train profile does not change with time, no underlying current exists, and the water surface is tension-free (uncontaminated). Water itself is assumed to be incompressible, inviscid (ideal fluid), and irrotational. Figure 2.5 shows the definition sketch of a wave train with $H$, $L$, $d$, and $\eta$, denoting wave height and length, water depth and surface elevation respectively. The governing equations of wave motion can be found in any classical text book on fluid mechanics (e.g. Sarpkaya and Isaacson 1981) and are given below for the sake of completeness.

![Figure 2.5: Wave Train Definition Sketch](image)

Defining a scalar function $\phi = \phi(x,z,t)$ so that the fluid velocity vector can be given by the gradient of $\phi$, it can be shown that based on the assumptions stated above, $\phi$, the so-called velocity potential, satisfies the two-dimensional Laplace equation:

$$\nabla^2 \phi = \frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial z^2} = 0$$  (2.30)
and is subject to the following boundary conditions at water surface and seabed:

\[
\frac{\partial \phi}{\partial z} = 0 \quad \text{at } z = -d \tag{2.31}
\]

\[
\frac{\partial \eta}{\partial t} + \frac{\partial \phi}{\partial x} \frac{\partial \eta}{\partial x} - \frac{\partial \phi}{\partial z} = 0 \quad \text{at } z = \eta \tag{2.32}
\]

\[
\frac{\partial \phi}{\partial t} + \frac{1}{2} \left[ \frac{\partial \phi}{\partial x} \left( \frac{\partial \phi}{\partial x} \right)' + \left( \frac{\partial \phi}{\partial z} \right)' \right] + g\eta = f(t) \quad \text{at } z = \eta \tag{2.33}
\]

\[
\phi(x, z, t) = \phi(x - ct, z) \tag{2.34}
\]

The boundary condition at the seabed states that the velocity vector has no component in vertical direction (Equation 2.31). The kinematics boundary condition at the water surface states that the velocity component normal to the water surface is equal to the velocity of water surface in that same direction (Equation 2.32). The dynamic boundary condition at the water surface states that the pressure along the surface is constant (equal to atmospheric pressure) (Equation 2.33). Equation (2.34) is based on the assumption of periodicity of the wave train where c=L/T denotes the wave celerity.

Given the wave height, period and the water depth, the question is what shape does the wave take and how to describe the water particles motion (displacements, velocities, and accelerations) throughout the flow. In solving the governing Laplace Equation (2.30) subject to boundary conditions explained in Equations (2.31-2.34), the following problems are encountered: the boundary conditions at the water surface are nonlinear and specified at a surface elevation \( \eta \), which is itself unknown. The various wave theories developed in
the past have tried to solve these problems with reasonable approximations. These include linear or Airy wave theory (also known as small amplitude wave theory), Stokes finite amplitude wave theories, Dean's stream function theory, and nonlinear shallow wave theories (such as Cnoidal wave theory). The question of suitability of a given wave theory for a particular application is a difficult one. One selection criteria is the amount of effort needed to produce the desired results. The more advanced the theory is, the more sophisticated the tools need to be to perform the analyses. Theoretical charts have been developed that show the ranges of best fit to the free surface boundary conditions for different wave theories (e.g., Figure 2.6). Experimental comparisons of different wave theories have not resulted in clear trends regarding the applicability of any particular wave theory (Sarpkaya and Isaacson, 1981). For the sake of simplicity and within the framework of a simulation analysis, only Airy small amplitude and Stokes finite amplitude wave theories are considered in this research.

The linear wave theory provides a first approximation of the wave motion. It is derived based on the assumption of relatively small wave heights, it is \( H \ll L, d \). The boundary conditions are satisfied at \( z = 0 \). Airy wave theory is very attractive to use for many engineering applications. It is simple and does not require computer analysis. There are approximations to linear wave theory for shallow water, intermediate depth and deep water ranges (see e.g. Sarpkaya and Isaacson, 1981). A practical approximation to Airy wave theory is the "depth-stretched" linear wave theory. In this approach, the water
surface is "stretched" to the wave crest elevation. The water particle kinematics are estimated according to the Airy small amplitude wave theory.

Based on a perturbation method, Stokes finite amplitude wave theories attempt to solve Equation (2.30) subjected to boundary conditions explained in Equations (2.31-2.34) more closely. However, like many other wave theories, convergence conditions put numerical limitations on wave heights in certain water depths. Worked by Skjelbreia and Hendrickson (1960) and Fenton (1985) on a fifth-order Stokes wave theory has found widespread interest and usage in engineering applications. Their formulations do not require extensive computer programming effort and is used in this research to develop a simplified load model.

2.3.2.2 Wave Directional Spreading

Real storm conditions include waves from multiple directions. Directional spreading of the waves reduces the loads acting on marine structures which are computed based on a two dimensional, long crested, regular wave grid propagating in a single horizontal direction. This load reduction is mainly due to change in water particle kinematics. Wave components from different directions can partially cancel each other. The effects of wave directionality have been investigated by many authors (e.g. Dean, 1977).

The detailed treatment of the subject is not within the scope of this work. In engineering practice, wave directional spreading effects are captured by a single wave kinematics modification factor. The actual water particle velocity is estimated by multiplying the
Figure 2.6: Regions of Applicability of Stream Function, Stokes V, and Linear Wave Theory (API, 1993a)
velocities based on a two-dimensional wave theory with a wave kinematics modification factor. Measurements indicate a range of 0.85 to 1.0 for highly directional seas during tropical storms to extra-tropical storm conditions (API, 1993).

2.3.2.3 Currents and Current Blockage

Currents can be a major contributor to total hydrodynamic forces acting on an offshore platform. In general, currents are generated in three ways: there are tidal, circulation, and storm generated currents. Tidal currents can be important in shallow waters of continental shelves (coastal regions and inlets). The Gulf Stream in the Atlantic Ocean and the Loop Current in the Gulf of Mexico are examples for large-scale circulation currents. Winds and pressure gradients during storms are the source of storm generated currents. These currents can be roughly estimated to have surface speeds of 1-3% of the one hour sustained wind speed during storms (API, 1993). The profile of storm generated currents is largely unknown and the subject of research. Here, in the MODU simulation model, three different types of current profile can be chosen. They are uniform, triangular and second order.

In determining the water particle kinematics due to currents, it should be recognized that, due to existence of the structure, the current is disturbed and its speed in the vicinity of the platform differs from that in the free field. Based on experimental test data, approximate current blockage factors for typical MODUs are given in API RP 2A (API, 1993). The actual current velocity in the vicinity of the structure is obtained by multiplying the free field current speed with the current blockage factor.
2.3.2.4 Wave and Current Loads

Morison, Johnson, O’Brien and Schaff (1950) proposed the following formulation for the force acting on a section of a pile due to wave motion:

\[ F = F_i + F_d = C_m \rho V \frac{du}{dt} + \frac{1}{2} C_s \rho A u |u| \]  \hspace{1cm} (2.35)

This formulation is widely known as Morison equation. According to Morison et al. (1950), this force is composed of two components: an inertia component related to the acceleration of an ideal fluid around the body, \( F_i \), and a drag component related to the steady flow of a real fluid around the body, \( F_d \). \( C_m \) is the so-called inertia coefficient, \( \rho \) is the mass density of fluid, \( V \) is the volume of the body and \( du/dt \) is the fluid acceleration. \( C_d \) is the so-called drag coefficient, \( A \) denotes the projected area of the body normal to the flow direction, and \( u \) is the incident flow velocity relative to pile.

Vortex shedding, drag and lift forces are all phenomena observed in real (viscous) fluids due to wake formation when the fluid passes a body. These phenomena do not exist in an ideal (inviscid) fluid. They have been the subject of comprehensive research for many decades and are now well understood and described for simple, idealized cases. In such cases, numerical computations are able to simulate these phenomena with reasonable degrees of accuracy. However, these programs are not yet efficient enough to be used by engineers and designers to calculate the forces on “real” marine structures.

Although extremely simple, the Morison’s equation has been used for many years by researchers and engineers to calculate the wave forces on “slender” marine structures. An
The important assumption implicit in the Morison equation is that the incident flow remains undisturbed in the vicinity of the body. This condition is satisfied when the body is small relative to the wave length. If the body is large relative to the wave length, the incident flow will not remain uniform and will be refracted due to presence of the body. In this case the refraction problem needs to be solved. For detailed treatment of the subject refer to Sarpkaya and Isaacson (1981). The refraction problem is not considered in this research since the platform dimensions are much smaller than the wave length in the extreme storm conditions.

The drag and inertia coefficients in Morison equation have empirical nature and depend on many factors including flow characteristics, shape and roughness of the body and its proximity to sea floor or free surface. One important flow parameter reflecting its uniformity is Keulegan-Carpenter (KC) number which is defined as:

$$ KC = \frac{UT}{D} $$

(2.36)

where $U$ and $T$ are the velocity amplitude and period of the oscillatory flow and $D$ is the diameter of the cylinder. Reynolds number, $Re$, is another important parameter that characterizes the flow regime reflecting its turbulence and is defined as:

$$ Re = \frac{UD}{v} $$

(2.37)

where $v$ denotes the fluid viscosity. Past field tests have indicated a large scatter in the values of drag and inertia coefficients when they are plotted against either the Reynolds number or the Keulegan-Carpenter number. This scatter is largely attributable to the
irregular nature of the ocean waves. Typical values for Reynolds and Keulegan-Carpenter numbers in extreme conditions are $Re > 10^6$ and $KC > 30$. For these ranges and based on experimental and field test data, mean drag and inertia coefficients are established for cylinders with smooth and rough surface (e.g. API, 1993).

2.3.3 A Simplified Load Model

Based on the background developed in the previous sections, a simplified load calculation model is developed and discussed in the following. Wind, current and wave forces are considered (Figure 2.7).

![Figure 2.7: The Simplified Load Model](image-url)
2.3.3.1 Wind Force

Given the wind velocity, the wind force acting on a moored floating MODU can be determined using Equation (2.38):

\[ F_w = C_w \sum (C_{A})V_w \]  

(2.38)

where:

- \( F_w \) = wind force, lb. (N)
- \( C_w = 0.0034 \text{lb/}(\text{ft}^2 \cdot \text{kt}) \) (0.615N\text{sec}^2/\text{m}^4)
- \( C_w \) = shape coefficient
- \( A \) = vertical projected area of each surface exposed to the wind, ft\(^2\) (m\(^2\))
- \( V_w \) = local wind speed, knots (m/sec)

The projected area exposed to the wind should include all columns, deck members, deck houses, trusses, crane booms, derrick substructure and drilling derrick as well as that portion of the hull above the water line.

2.3.3.2 Current Forces

Current forces are normally treated as steady state forces in a mooring analysis. Current force acting on semisubmersible hulls can be calculated as:

\[ F_u = C_u(C_uA_u + C_A)U_i \]  

(2.39)

where:

- \( F_u \) = current force, lb(N)
\[ C_\alpha = \text{current force coefficient for semisubmersible hulls} \]
\[ = 2.85 \text{lb} / (\text{ft}^2 \cdot \text{kt}) (515.62 \text{N sec}^2 / \text{m}^4) \]

\[ C_d = \text{drag coefficient (dimensionless)} \]
\[ = 0.6 \text{ for circular members; 1.0 for members having flat surfaces.} \]

\[ A_t = \text{summation of total projected areas of all cylindrical members below the waterline. ft}^2(\text{m}^4) \]

\[ A_f = \text{summation of projected areas of all members having flat surfaces below the waterline. ft}^2(\text{m}^4) \]

### 2.3.3.3 Wave Force

Interactions between ocean waves and a floating vessel results in forces acting on the vessel, which can be conveniently split into three categories (Figure 2.8):

1. **First-order forces** that oscillate at the wave frequencies. They induce first-order motions which are also known as high frequency or wave frequency motions.

2. **Second order forces** with frequencies below wave frequencies. They induce second order motions which are also known as low frequency motions.

3. **Steady component** of the second order forces which is known as mean wave drift force.

- **Wave Frequency MODU Motions**

The motions of the MODU at the frequency of the waves is an important contribution to the total mooring system loads, particularly in shallow water. These wave frequency
motions can be obtained from regular or random wave model test data, or computer analysis using either time or frequency domain techniques. The method used in this work is based on the widely used Morison Equation (Eq. 2.35).

![Wave Force Components](image)

Figure 2.8 Wave Force Components

The total hydrodynamic force per unit length, $F$, is comprised of a drag force, $F_d$, and an inertia force, $F_i$:

$$F = F_d + F_i$$  \hspace{1cm} (2.40)

where,

$$F_d = C_d \frac{\rho}{2} u |u| $$  \hspace{1cm} (2.41)

$$F_i = K_i |u| $$

and,

$$F_i = C_i \frac{\rho \pi D^4}{4} a$$  \hspace{1cm} (2.42)

$$F_i = K_i (a)$$
The total lateral force can be calculated by integrating the local forces over the entire structure. Due to 90 degree phase angle difference between the maximum drag and inertia force components and the relatively large dimensions of a typical MODU type platform, the wave force is inertia force dominant. That means that at the time the inertia forces acting on the platform reach a maximum value the drag forces are relatively small and hence are neglected in this work.

All of the structure elements are modeled as equivalent vertical cylinders (Mortazavi, Bea, 1995). Appurtenances (conductors, boat landings, risers) are modeled in a similar manner. For wave crest elevations that reach the lower decks, the horizontal hydrodynamic force acting on the lower decks are computed based on the projected area of the portions of the structure that would be able to withstand the high pressures.

Airy wave theory and Stokes 5th theory are used to calculate wave kinematics:

a. Airy Wave Theory

For uni-directional (long-crested) waves, water particle horizontal velocities, \( u_w \) and accelerations, \( a_w \) are

\[
u_w = \left( \frac{\pi H}{T} \right) e^{\nu} \cos(\theta) \tag{2.43}
\]

and,

\[
a_w = \left( 2\pi t \frac{H}{T} \right) e^{\nu} \sin(\theta) \tag{2.44}
\]
where \( k \) is the wave number \( (k = 2\pi / L) \), \( z \) is the vertical coordinate which is zero at the still water level and positive upward, and \( \theta \) is the wave phase angle \( (\theta = kx - \omega t) \), \( \omega \) is the wave circular frequency, \( \omega = 2\pi / T \), \( x \) is the horizontal coordinate measured from the wave crest, and \( t \) is the time coordinate.

b. Stokes' 5th Theory

For Stokes theory, using equations given by Skjelbreia and Hendrickson (1961) and Fenton (1985), a computer program was developed to determine the wave kinematics (Preston, 1994). Given the wave height \( H \), period \( T \) and water depth \( d \), the vertical profile of maximum horizontal velocities and accelerations beneath the wave crest are estimated as:

\[
\frac{U}{C} = K_s \sum n \Phi_n \cosh(nks) \tag{2.45}
\]

\[
\frac{a}{\omega c} = K_s \sum n \Phi_n \cosh(nks) \tag{2.46}
\]

where \( K_s \) is a coefficient that recognizes the effects of directional spreading and wave irregularity on the Stokes wave theory based velocities. \( k \) is the wave number and \( s \) is the vertical coordinate counting positive upward from the sea floor. \( c \) is the wave celerity and given as:

\[
\frac{c^2}{g} = \frac{\tanh(kd)}{kd} \left[ 1 + \lambda^2 C_1 + \lambda C_2 \right] \tag{2.47}
\]

The crest elevation \( \eta \) is estimated as:
\[ k \eta = \sum_{n=1}^{\infty} \eta_n C_n \]  

(2.48)

\( \Phi \) and \( \eta \) are given functions of \( \lambda \) and \( kd \). \( C_n \) are known functions of \( kd \) only, given by Skjelbreia and Hendrickson (1961). The wave number \( k \) is obtained by implicitly solving equation given by Fenton (1985):

\[
\frac{2\pi}{T(gk)^n} - C_n - \left(\frac{kH}{2}\right)^n C_n - \left(\frac{kH}{2}\right)^n C_n = 0
\]  

(2.49)

The parameter \( \lambda \) is then calculated using the equation given by Skjelbreia and Hendrickson(1961):

\[
\frac{2\pi d}{gT} = \frac{d}{L} \tanh(kd)[1 + \lambda'C_1 + \lambda'C_1]
\]  

(2.50)

- **Mean Wave Drift Force**

The mean wave drift force is induced by the steady component of the second order wave forces. The determination of mean drift force requires motions analysis computer programs or model tests. Design curves for estimating mean wave drift forces for semisubmersibles are provided in API (1991, 1994) (Figure 2.9). The curves are applicable to typical MODU type vessels.

- **Low Frequency Vessel Motions**

Low frequency motions are induced by the low frequency component of the second order wave forces which in general are quite small compared to the first order forces. Sometimes the second order forces are amplified through resonance into motions which can become very large and neglecting low frequency motions can provide non-